

# ROCKING DAMAGE-FREE STEEL COLUMN BASE WITH FRICTION DEVICES

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## 1. ABSTRACT

Earthquake resilient steel frames, such as frames with self-centering connections or frames with passive energy dissipation devices, have been extensively studied during the past decade but little attention has been paid to their column bases. The paper presents a rocking damage-free steel column base, which uses post-tensioned (PT) high strength steel bars to control rocking behavior and friction devices to dissipate seismic energy. Contrary to conventional steel column bases, the rocking column base exhibits monotonic and cyclic moment-rotation behaviors that are easily described using simple analytical equations. Analytical equations are provided for different cases including yielding or loss of post-tensioning in the PT bars and their efficiency is compared with numerical results from a three-dimensional non-linear finite element (FE) model in ABAQUS. Moreover, a simplified model is developed in OpenSees to evaluate how the use of the rocking column base affects the global behavior of a self-centering moment-resisting frame. Nonlinear dynamic analyses show that the rocking column base fully protects the first story columns from yielding and eliminate the first story residual story drift.

## 2. INTRODUCTION

Conventional seismic-resistant structures, such as steel moment resisting frames (MRFs), are designed to experience inelastic deformations under strong earthquakes [1]. Inelastic

deformations result in damage and residual story drifts, and inevitably, in high repair costs and disruption of the building occupation. The latter socio-economic risks highlight the need for minimal-damage structures, which can reduce both repair costs and downtime. Many earthquake-resilient steel frame typologies have been extensively studied during the last decade but little attention has been paid to their column bases (CBs).

Conventional steel CBs consist of an exposed steel base plate supported on grout and secured to the concrete foundation using steel anchor rods. In terms of strength, CBs are typically designed as full-strength so that plastic hinges are developed in the bottom end of the first story columns [1], [2]. Apart from the fact that plastic hinges in the columns induce non-repairable damage, this design approach needs very strong CBs with adequate over-strength. Alternatively, Eurocode 8 allows the design of partial-strength CBs designed to develop plastic deformations. Such design philosophy however needs the knowledge of the plastic rotation capacity of the CB, which is difficult to predict. Most importantly, field observations after strong earthquakes confirmed the susceptibility of CBs to difficult-to-repair damage such as concrete crushing, anchor rod fracture and base plate yielding [3].

Several alternative CBs have been proposed recently with the goal of overcoming the shortcomings of conventional CBs. Some of them used steel bars as re-centering system [4], while others used replaceable bolts. An alternative approach has been presented in [5] where the column base was equipped with asymmetric friction devices.

This paper presents an innovative rocking damage-free steel CB, recently proposed by [6], which uses post-tensioned (PT) high strength steel bars to control rocking behavior and friction devices (FDs) to dissipate seismic energy. Contrary to conventional steel CBs, the proposed CB has monotonic and cyclic moment-rotation behaviors that are easily described using simple analytical equations. Analytical equations are provided for different cases including structural limit states that involve yielding or loss of post-tensioning in the PT bars. A step-by-step design procedure is developed, which ensures damage-free behavior, self-centering capability, and adequate energy dissipation capacity for a predefined target CB rotation. A three-dimensional non-linear finite element (FE) model is developed in ABAQUS [7], which is used to validate the efficiency of the moment-rotation analytical equations and the design procedure. A simplified model is developed in OpenSees [8] to evaluate how the proposed CB affects the global behavior of a steel frame. A prototype steel building is designed as a self-centering moment-resisting frame (SC-MRF) with conventional or rocking CBs. Nonlinear dynamic analyses show that the rocking CB fully protects the 1<sup>st</sup> story columns from yielding and eliminate the 1<sup>st</sup> story residual story drift.

### 3. ROCKING DAMAGE-FREE COLUMN BASE

Fig. 1(a) shows the proposed rocking damage-free CB. A thick steel plate with rounded edges is welded on the bottom of a circular hollow steel section. The rounded edges help the CB to avoid damage during rocking. Four PT high strength steel bars are symmetrically placed around the center of the CB to control the rocking behavior. The PT bars are anchored to the bottom of the foundation and to a thick plate welded on the top of the hollow steel section (*i.e.* anchor plate in Fig. 1(a)). FDs are placed to the four sides of the CB to provide energy dissipation. The FDs consist of two external steel plates bolted to the base plate and drilled with aligned rounded holes; an internal steel plate with inclined slotted holes, welded to the circular hollow section; and two plates of brass material in the interface. Rocking of the CB results in sliding of the internal plate with respect to the brass and external plates, and thus, in energy dissipation due to friction. Pre-tensioned bolts are used to tune the friction force in the FDs. More details are reported in [6].

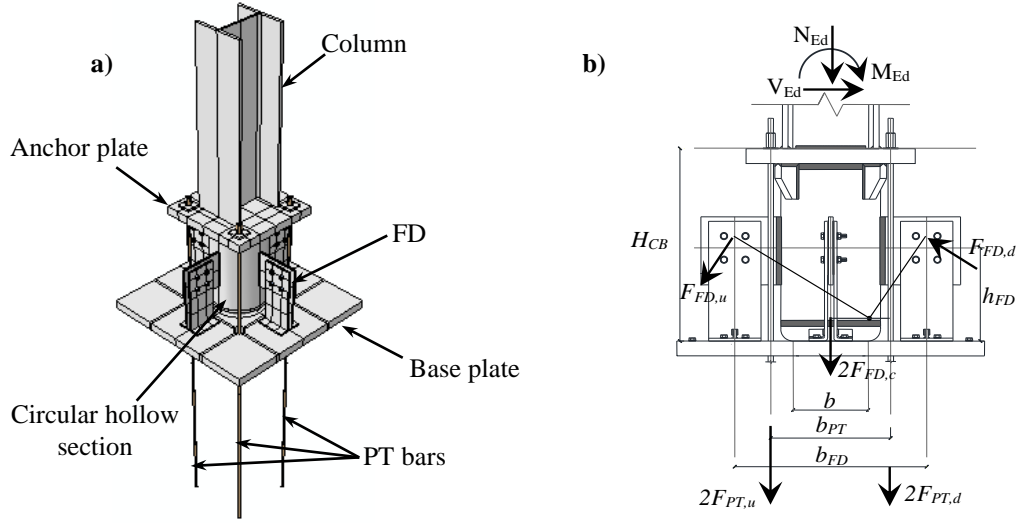


Fig. 1. a) 3D view of the proposed column base; b) fundamental dimensions and forces in the FDs and PT bars at the onset of rocking for loading from left to right

Fig. 1(b) shows the dimensions of the CB that control the moment-rotation behavior due to rocking, *i.e.*  $b$  = dimension of the contact surface;  $b_{PT}$  = distance among the PT bars;  $b_{FD}$  = distance among the FDs; and  $h_{FD}$  = distance of the FDs from the base plate. Fig. 1(b) shows the forces in the CB's components when it is at the onset of rocking with respect to its right edge under the effect of the internal axial force ( $N$ ), shear force ( $V$ ), and bending moment ( $M$ ). In Fig. 1(b),  $F_{PT,u}$  and  $F_{PT,d}$  are the forces in the PT bars while  $F_{FD,u}$ ,  $F_{FD,d}$  and  $F_{FD,c}$  are the forces in the FDs. The subscripts  $u$  and  $d$  denote whether the point of application of these forces will move upwards or downwards during rocking. The subscript  $c$  denotes the force in each of the two central FDs. The lever arms of the forces with respect to the center of rotation  $z_{PT,u}$ ,  $z_{PT,d}$ ,  $z_{FD,u}$ ,  $z_{FD,c}$ ,  $z_{FD,d}$  are easily derived from the main dimensional parameters [6]. The moment contribution of the axial force,  $N$ , is given by

$$M_N = N \cdot b/2 \quad (1)$$

The forces in the PT bars are functions of the rotation,  $\theta$ , of the CB and are given by

$$F_{PT,u} = T_{PT} + K_{PT} \cdot z_{PT,u} \cdot \theta \quad \text{for } \theta \leq \theta_{PT,u,y} \quad (2.a)$$

$$F_{PT,d} = T_{PT} - K_{PT} \cdot z_{PT,d} \cdot \theta \quad \text{for } \theta \leq \theta_{PT,d,f} \quad (2.b)$$

where  $T_{PT}$  is the initial post-tensioning force of each PT bar;  $K_{PT} = E_{PT}A_{PT}/L_{PT}$  the stiffness of each PT bar;  $E_{PT}$  the Young's modulus;  $A_{PT}$  the cross-sectional area;  $L_{PT}$  the length of each PT bar;  $\theta_{PT,u,y}$  the rotation at which the PT bars (in position  $u$ ) yield; and  $\theta_{PT,d,f}$  the rotation at which the force of the PT bars (in position  $d$ ) becomes zero, *i.e.* when loss of post-tensioning occurs. The PT bars should be designed to avoid either yielding or loss of post-tensioning for a target rotation  $\theta_T$ . The moment contribution of the PT bars is given by

$$M_{PT}(\theta) = 2 \left[ T_{PT} (z_{PT,u} - z_{PT,d}) + K_{PT} (z_{PT,u}^2 + z_{PT,d}^2) \theta \right] \quad \text{for } \theta \leq \theta_T \quad (3)$$

The friction force,  $F_{FD,i}$ , in each FD is given by

$$F_{FD,i} = \mu_{FD} \cdot n_b \cdot N_b \quad \text{with } i = u, c, d \quad (4)$$

where  $\mu_{FD}$  is the friction coefficient of the surfaces in contact;  $n_b$  the number of bolts; and  $N_b$  the bolt preloading force. Therefore, the moment contribution of the FDs is given by

$$M_{FD} = 2 \cdot F_{FD} \left( z_{FD,u} + 2 \cdot z_{FD,c} + z_{FD,d} \right) \quad (5)$$

Fig. 2(a) shows the moment contributions of the axial force,  $M_N$ ; the PT bars,  $M_{PT}$ ; and the FDs,  $M_{FD}$ . The decompression moment,  $M_E$ , and the moment at the onset of rocking,  $M_D$ , are given by

$$M_E = M_N + M_{PT,0} \quad \text{and} \quad M_D = M_E + M_{FD} \quad (6)$$

where  $M_{PT,0}$  = moment provided by the PT bars at zero rotation (*i.e.*  $\theta = 0.0$  in Eq. (3)). The rotational stiffness contribution of the PT bars is given by

$$S_{PT} = 2K_{PT} \left( z_{PT,u}^2 + z_{PT,d}^2 \right) \quad (7)$$

Therefore, the moments of the cyclic  $M$ - $\theta$  behavior of the CB in Fig. 2(b) are given by

$$M_1 = M_D = M_N + M_{PT,0} + M_{FD} \quad (8.a)$$

$$M_2 = M_D + S_{PT}\theta_2 \quad (8.b)$$

$$M_3 = M_D + S_{PT}\theta_2 - 2M_{FD} \quad (8.c)$$

$$M_4 = M_D - 2M_{FD} \quad (8.d)$$

To ensure that the CB provides full self-centering capability,  $M_4 \geq 0$  or  $M_E \geq M_{FD}$ .

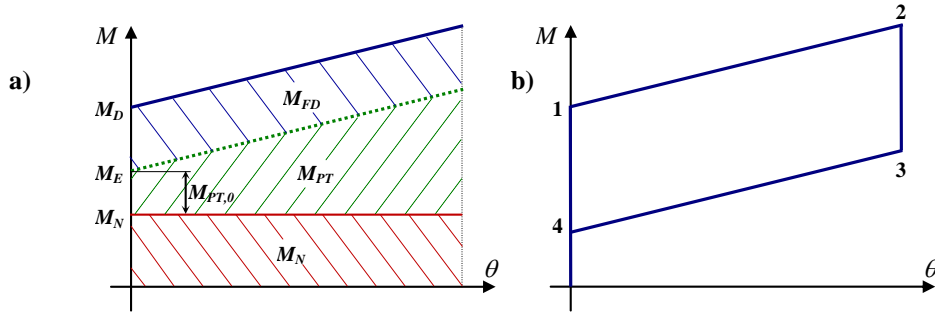


Fig. 2. Moment-rotation behavior of the column base. a) moment contribution of the axial force,  $M_N$ ; of the PT bars,  $M_{PT}$ ; and of the FDs,  $M_{FD}$ ; b) hysteretic behavior

#### 4. CASE STUDY ROCKING COLUMN BASE

The novel CB is designed for a HEB 300 and an axial forces of  $N_{Ed,G} = 537.8$  kN with a corresponding plastic moment resistance  $M_{N,Rd} = 308.9$  kNm. A  $\theta_T = 0.023$  rad is used for the design of the CB. A circular hollow section with 323.9 mm diameter and 40 mm thickness is adopted. A circular steel plate with circular rounded edges with a radius of 40 mm is welded at the bottom of the hollow section, and hence,  $b = 243.9$  mm. The distance among the PT bars,  $b_{PT}$ , is 390 mm. The PT bars material properties are  $E_{PT} = 205$  GPa and  $f_{y,PT} = 900$  MPa. The design is based on the methodology proposed in [6] and on the following equations

$$\kappa = \frac{1}{2A_{PT}f_{y,PT}(z_{PT,u} - z_{PT,d})} \left[ \frac{M_T - 2 \frac{E_{PT}A_{PT}}{L_{PT}} (z_{PT,u}^2 + z_{PT,d}^2) \theta_T}{1 + \frac{1}{\alpha_{sc}}} - N_{Ed,G} \frac{b}{2} \right] \quad (9.a)$$

$$\kappa \leq 1 - \frac{E_{PT} \cdot z_{PT,u} \cdot \theta_T}{f_{y,PT} \cdot L_{PT}} = \kappa_{\max} \quad \text{and} \quad \kappa \geq \frac{E_{PT} \cdot z_{PT,d} \cdot \theta_T}{f_{y,PT} \cdot L_{PT}} = \kappa_{\min} \quad (9.b)$$

where  $M_T = M_{N,Rd}/\gamma_T$  is the moment at  $\theta_T$  that, through the capacity coefficient  $\gamma_T$ , protects the column from yielding (i.e.  $\gamma_T = 1.4$ ) and  $\alpha_{sc} = M_E/M_{FD}$  is a design parameter that controls self-centering capability (i.e.  $\alpha_{sc} = 1.3$ ).  $A_{PT}$ ,  $L_{PT}$  and  $\kappa$  are the design variables of the problem and  $\kappa = \sigma_{PT}/f_{y,PT}$  is the stress ratio in the strands ( $\sigma_{PT}$  and  $f_{y,PT}$  are respectively the stress and the yield stress of the strands) for the definition of the initial post-tensioning force. Fig. 3(a) shows the variation of  $\kappa$  with respect to  $L_{PT}$  for different  $d_{PT}$  values.

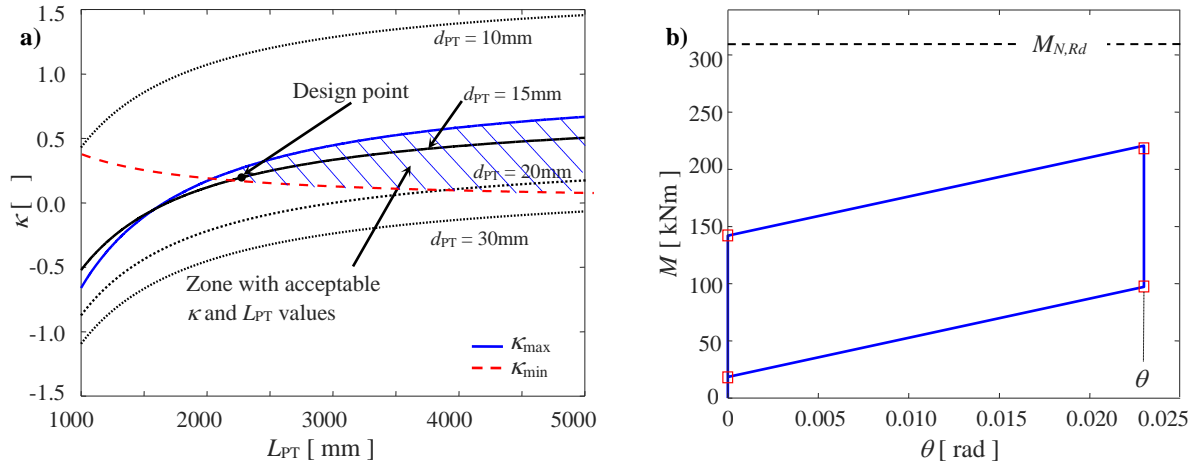


Fig. 3. a) Variation of  $\kappa$  with respect to  $L_{PT}$  for different  $d_{PT}$  values; (b) moment-rotation behavior of the column base

Any pair of  $\kappa$  and  $L_{PT}$  within the highlighted acceptable zone can be selected but the optimum design is close to the intersection of the  $\kappa_{min}$  and  $\kappa_{max}$  curves. In this example,  $d_{PT} = 15$  mm,  $L_{PT} = 2240$  mm,  $\kappa = 0.189$  ( $T_{PT} = 30$  kN),  $\theta_{PT,u,y} = 0.0252$  rad and  $\theta_{PT,d,f} = 0.0255$  rad. Fig. 3(b) shows the moment-rotation behavior for the column base. The decompression moment,  $M_E$ , is equal to 80.3 kNm, the moment at the onset of rocking,  $M_D$ , is equal to 142.0 kNm, and the moment provided by the FDs,  $M_{FD}$  is equal to 61.7 kNm. FDs are introduced on the four sides of the column base; the relevant dimensions are  $b_{FD} = 623.9$  mm and  $h_{FD} = 315$  mm. The friction coefficient  $\mu_{FD}$  is equal to 0.15. Four M12 class 10.9 bolts are preloaded at 45 kN to achieve the required friction force.

## 5. FINITE ELEMENT MODELS FOR THE COLUMN BASE

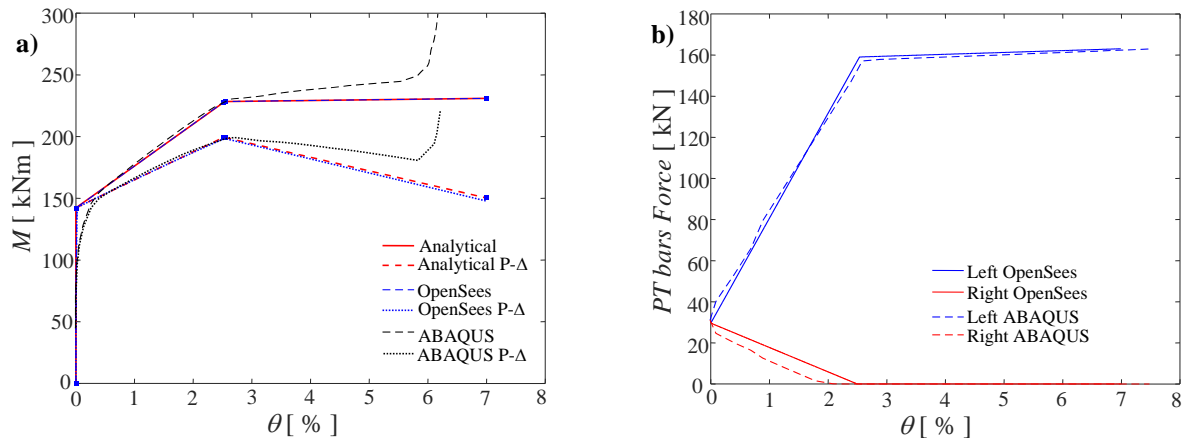


Fig. 4. (a) Monotonic moment-rotation behavior of the column base; (b) forces in the PT bars for analysis with P- $\Delta$  effects (results are practically the same for analysis without P- $\Delta$  effects).

Simplified and detailed FE models are developed in OpenSees [8] and ABAQUS [7], respectively, to simulate the behavior of the rocking CB. Fig. 4(a) shows the monotonic moment-rotation curve for rotations up to 0.07 rad. Fig. 4(b) shows the forces in the PT bars (loading direction from left to right) as functions of the rotation of the CB. It is seen that yielding of the left PT bars and loss of post-tensioning of the right PT bars take place almost simultaneously at rotations close to the target one (*i.e.* 0.023 rad). Excellent agreement of the results is also obtained for cyclic analyses confirming the effectiveness of the design procedure. Additional details are reported in [6].

## 6. EFFECT OF ROCKING COLUMN BASES ON GLOBAL SEISMIC RESPONSE

A 5-story, 5-bay by 3-bay steel building with two perimeter SC-MRFs with PT beam-column connections is used as case study. The design basis earthquake (DBE) is expressed by the Type 1 elastic response spectrum of Eurocode 8 [1] with PGA equal to 0.35g and ground type B. The maximum credible earthquake (MCE) is assumed to have intensity equal to 150% the DBE intensity. The SC-MRF has been modeled in OpenSees [8] in a similar way as in [9] for cases with conventional and with rocking CB. The SC-MRF with conventional CBs has  $T_I = 0.94$  sec, while the SC-MRF with the rocking CBs has  $T_I = 0.867$  sec. The difference is due to the shorter flexible length of the 1<sup>st</sup> story columns in the case with rocking CBs. Ten earthquake ground motions are used for nonlinear dynamic analyses. They are scaled to the DBE and MCE seismic intensities which is described by the spectral acceleration,  $S_a$ , at  $T_I$ , while the inherent damping ratio is 3%. The results show that the SC-MRF with conventional CBs experiences appreciable residual 1<sup>st</sup> story drifts due to 1<sup>st</sup> story column yielding. Such residual drifts reach values close to 0.5% under individual earthquake ground motions (*i.e.* a critical value as the limit beyond which repair of a steel building may not be economically viable). The use of the rocking CBs eliminates the 1<sup>st</sup> story residual drift. Fig. 5(a) shows the 1<sup>st</sup> story drift time histories of the SC-MRFs for a specific earthquake ground motion scaled at the DBE intensity. The two SC-MRFs experience similar peak 1<sup>st</sup> story drifts but the residual 1<sup>st</sup> story drifts are minimized in the case with the rocking CBs. For the same ground motion, Fig. 5(b) compares the stress-strain hysteresis in the flanges of one of the 1<sup>st</sup> story columns. The SC-MRF with conventional CBs experience plastic deformations and damage, while the use of rocking CBs fully protects the columns from yielding.

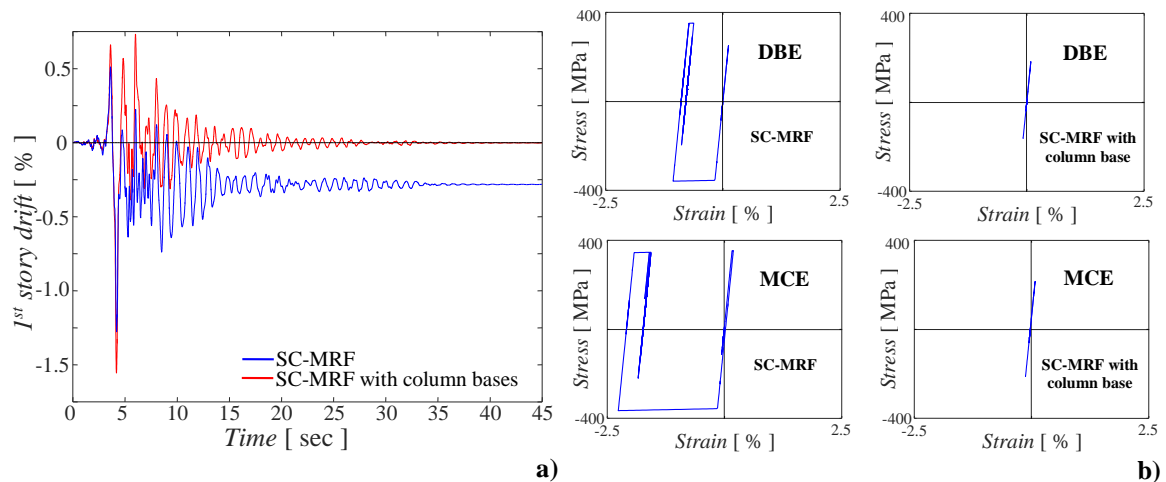


Fig. 5. a) 1<sup>st</sup> story drift time histories for a specific ground motion scaled at the DBE intensity; (b) Stress-strain hysteresis in the flanges of one of the 1<sup>st</sup> story columns of the SC-MRFs for a specific ground motion scaled at the DBE and MCE intensities

## 7. CONCLUSIONS

A rocking damage-free steel column base has been presented. The column base uses post-tensioned (PT) high strength steel bars to control rocking behavior and friction devices to dissipate seismic energy [6]. Contrary to conventional steel column bases, the rocking column base has monotonic and cyclic moment-rotation behaviors that are easily described using simple analytical equations. Analytical equations are provided for different cases including structural limit states that involve yielding or loss of post-tensioning in the PT bars. A step-by-step design procedure was presented, which ensures damage-free behavior, self-centering capability, and adequate energy dissipation capacity under a predefined target column base rotation. A three-dimensional non-linear finite element (FE) model of the column base was developed in ABAQUS. The results of the FE simulations validate the accuracy of the moment-rotation analytical equations and demonstrate the efficiency of the design procedure. A simplified model for the column base was developed in OpenSees and validated against the detailed FE models. A prototype steel building was designed as a self-centering moment-resisting frame with conventional or rocking column bases. Nonlinear dynamic analyses show that the rocking column base fully protects the 1<sup>st</sup> story columns from yielding and eliminate the 1<sup>st</sup> story residual drift without any detrimental effect on peak story drifts.

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## **ΠΕΡΙΛΗΨΗ**

Σημαντική έρευνα έχει συντελεστεί για την ανάπτυξη αντισεισμικών μεταλλικών πλαισίων ικανών να αποφύγουν τις πλαστικές παραμορφώσεις και βλάβες στο φέρων οργανισμό (δοκοί, υποστυλώματα, χιαστοί σύνδεσμοι, συνδέσεις δοκού-υποστυλώματος). Παρόλα αυτά ελάχιστες εργασίες έχουν εστιάσει στις συνδέσεις έδρασης των υποστυλωμάτων. Η εργασία παρουσιάζει νέα σύνδεση έδρασης μεταλλικών υποστυλωμάτων μηδενικής βλάβης. Η σύνδεση βασίζεται στο λικνισμό ισχυρού κοντού υποστυλώματος με κατάλληλη διαμόρφωση του ελάσματος βάσης για την αποφυγή συγκέντρωσης τάσεων. Προεντεταμένοι μεταλλικοί ράβδοι προσδίδουν αυτο-επαναφερόμενη συμπεριφορά ενώ η απόσβεση ενέργειας επιτυγχάνεται μέσω συνδυασμού μεταλλικών ελασμάτων και προεντεταμένων κοχλιών για την ανάπτυξη τριβής. Η εργασία παρουσιάζει τρισδιάστατα μοντέλα πεπερασμένων στοιχείων στο πρόγραμμα ABAQUS για τη προτεινόμενη σύνδεση έδρασης και σύγκριση των αποτελεσμάτων τους με απλές αναλυτικές εξισώσεις περιγραφής της υστερητικής συμπεριφοράς. Τα αποτελέσματα των αναλύσεων και η προαναφερθείσα σύγκριση δείχνουν ότι η σύνδεση δε παρουσιάζει βλάβη και ότι η υστερητική συμπεριφορά της μπορεί να προσομοιωθεί ικανοποιητικά με απλές αναλυτικές εξισώσεις. Επιπλέον, σεισμικές αναλύσεις στο πρόγραμμα OpenSees δείχνουν ότι μεταλλικά πλαίσια που χρησιμοποιούν τη προτεινόμενη σύνδεση βάσης αποφεύγουν πλήρως τις πλαστικές παραμορφώσεις στα υποστυλώματα και ελαχιστοποιούν τη παραμένουσα γωνιακή παραμόρφωση του πρώτου ορόφου.